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March 5, 2010

U.S. Nuclear Regulatory Commission
Attn: Document Control Desk
Washington, D.C., 20555-001

Subject: Duke Energy Carolinas, LLC
Oconee Nuclear Station, Units 1, 2, and 3
Renewed Facility Operating License, DPR-38, DPR-47, and DPR-55
Docket Numbers 50-269, 50-270, and 50-287
Oconee External Flood, Response to Request for Additional Information (RAI)

References:

1. NRC Letter From Joseph G. Giitter to Dave Baxter, "Information Request Pursuant to 10 CFR 50.54(f) Related to External Flooding Including Failure of the Jocassee Dam, at Oconee Nuclear Station, Units 1, 2, and 3 (Oconee) (TAC Nos. MD8224, MD8225, and MD8226)," dated August 15, 2008
2. Duke Energy Letter From Dave Baxter to NRC Document Control Desk, "Oconee External Flood Analyses and Associated Corrective Action Plan", dated November 30, 2009
3. NRC Letter From Joseph G. Giitter to Dave Baxter, "Evaluation of Duke Energy Carolinas, LLC (Duke Energy), November 30, 2009, Response to Nuclear Regulatory Commission (NRC) Letter dated April 30, 2009, Related to External Flooding at Oconee Nuclear Station, Units 1, 2, and 3 (Oconee) (TAC Nos. ME3065, ME3066, and ME3067)," dated January 29, 2010
4. Duke Energy Letter From Dave Baxter to NRC Document Control Desk, "Oconee External Flood Analyses, Response to Request for Additional Information (RAI)," dated February 8, 2010

On August 15, 2008, the NRC issued a request for information pursuant to Title 10 of the *Code of Federal Regulations* (10 CFR) Part 50, Section 50.54(f) regarding the protection against external flooding at Oconee including a postulated failure of the Jocassee Dam (Reference 1). As part of its response, Duke Energy provided details of Oconee external flood analyses and associated corrective action plans in a letter dated November 30, 2009 (Reference 2).

On February 3, 2010, Duke Energy received Reference 3 requesting additional information (RAI) within 30 days and a written notification within 5 days if the RAI response cannot be provided within 30 days. Subsequently, Duke Energy determined that the information requested by some RAI questions required further analysis that could not be completed within 30 days. Thus, Duke Energy notified the NRC of that determination by letter dated February 8, 2010 (Reference 4).

The purpose of this letter, its attachments and enclosure is to provide a response to RAI questions 2, 3, 5, 6 and 8 and partial responses to RAI questions 1, 4, and 7. Duke Energy plans to provide additional information regarding questions 1 and 4 by June 2010. Furthermore, we plan to provide additional information regarding question 7 following additional discussions with the NRC about the request, availability of needed information, and timing needs.

This letter, its attachments, and enclosure contain security sensitive information. As such, Duke Energy hereby requests the NRC withhold this letter, its attachments, and enclosure from public disclosure pursuant to 10 CFR 2.390 (d)(1), "Public inspections, exemptions, requests for withholding."

There are no new regulatory commitments contained in this letter, its attachments or enclosure. If you have any questions on this matter, please contact Jeff Thomas, Fleet Regulatory Compliance Manager, (704) 382-3438, or Bob Meixell, Oconee Regulatory Compliance, (864) 873-3279.

Sincerely,

Dave Baxter, Vice President
Oconee Nuclear Station

Attachments:

Attachment 1; RAI Responses
Attachment 2; Oconee Nuclear Site Topographical Drawing
Enclosure 1; Oconee Nuclear Site Topographical Drawing [Electronic Copy (CD)]

Dave Baxter affirms that he is the person who subscribed his name to the foregoing statement, and that all the matters and facts set forth herein are true and correct to the best of his knowledge.

[Executive Name]

Subscribed and sworn to me: 3/5/2010
Date

Ruth H Joyner
Notary Public

My Commission Expires: 6/15/2016
Date

SEAL

~~This letter, its attachments, and enclosure contain security sensitive information.~~
~~Withhold from Public Disclosure under 10 CFR 2.390 (d)(1)~~

bc w/attachment 1:

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1. **Question:**

Justify the assumptions used for parameters (breach dimension, breach time, and breach location) associated with the Jocassee Dam, Keowee Main Dam, Keowee West Saddle Dam, Intake Dike, and the Little River Dam. Also include the assumptions associated with the operation and capacity of the turbines and discharge gates for the Jocassee Dam. Specifically, describe how the values selected for each parameter represent a conservative value.

Response: (interim)

This response contains a technical discussion of the selected parameters used in the Hydrologic Engineering Center-River Analysis System (HEC-RAS) Cases 1, 2, and 3 as presented to the Nuclear Regulatory Commission (NRC) on October 28, 2009. More specifically, for this RAI response, the HEC-RAS run and 2-dimensional (2-D) model used to assess potential site flooding is Case 2. Additional modeling and runs are in progress to complete the response to this RAI. Also, further modeling and runs may be required to establish the appropriateness of the corrective action plan.

Justification of the selected parameters and the variation of those parameters used in the sensitivity analysis are also included. A portion of this RAI response was provided to the staff in our November 30, 2009 letter. It is being resubmitted here for completeness. The intent of the sensitivity analysis was to determine those parameters that would have a significant effect on the results of the HEC-RAS inundation analysis following a postulated failure of the Jocassee Main Dam. Significance in this case is defined as affecting the potential water level at the Oconee site. The significance of the effects was determined by varying those selected parameters and gauging the results. A total of 98 HEC-RAS runs were evaluated for sensitivity of the parameters prior to the generation of the 3 runs known as Cases 1, 2, and 3.

The parameters for the postulated failure of Jocassee Dam during a 'sunny day' scenario are presented below. The technical basis for the selection of those parameters is presented as well. Based on our research, derivation of dam failure parameters appropriate for a rock-fill dam like Jocassee for this scenario yields a broad range of values. The available research information is limited. Based on our hydraulic models for this event, we do expect to see overtopping of the Keowee impoundment structures during the worst case Jocassee Dam failure. If this overtopping causes Keowee impoundment structures to fail (cascading dam scenario), then available research information is even more minimal for this case.

Understanding the availability of research information, guidelines, and or standards for such scenarios, Duke Energy has sought to take a reasonable and conservative approach to this assessment. In general, research work by Froehlich guided the selection of many of the parameters providing the physical and material parameters of our structures were compatible with those generated using methodology from his work. As shown below, conservatism is demonstrated by using his work to determine breach sizes for the Jocassee Dam and then comparing the computed peak breach flows from the HEC-RAS runs verses his estimated flows. For consistency, Froehlich's work was used as well for the Keowee impoundment structures. We do understand this may not fully represent the extreme failures that may occur for these structures during a cascading dam failure event. However, in the absence of observed technical information for similar events, this was the best available information to determine the expected failure parameters.

The investigation and sensitivity assessment to determine the appropriate failure parameters for the Keowee impoundment structures is continuing. Additional research is underway regarding the 1975 Chinese cascading dam failures to find available information providing insights that may be beneficial to assure we characterize the failures appropriately. Also, additional 2-D evaluations are in progress to determine the sensitivity of the failures of the Keowee impoundment structures. (The HEC-RAS run evaluations give us a 'big picture' look at the impact, but the 2-D evaluations give us a more detailed look at the impact as it directly relates to the Oconee site.) The additional work is being investigated from two perspectives: Determine a technical basis for the parameters or assess the sensitivity of the parameters in the absence of a technical basis.

The additional 2-D assessments planned will determine the impact of faster breach times for the Keowee Main Dam as well as other Keowee impoundment structures having an influence on the resulting tailrace elevation below the Keowee Dam. These runs would assess Keowee impoundment structure failure time at lower bound values and possible midpoints for sensitivity. Based on these results, varying failure times in combination between the structures may be warranted. This additional scope of work will provide updated information and may affect the parameter selection and justification for the Keowee impoundment structures. Therefore, the information provided below is considered an interim response. A final response to this RAI will be submitted in June 2010.

Jocassee Parameters

In addition to the selection of the breach size and time to failure parameters, other parameters were selected to support the analysis. For example, since the issue regards a postulated 'sunny day' failure of the Jocassee Dam, only those failure modes that could occur were considered, given the design and construction of Jocassee. Since Jocassee is designed to pass a Probable Maximum Flood (PMF) without overtopping the main dam, overtopping failures were not considered. Likewise seismic induced failures were not considered since Jocassee is seismically designed.

It was the opinion of the dam experts consulted by Duke Energy (RAC Engineers & Economists (Utah State University) and HDR/DTA) that the predominant failure mode affecting the structural stability of the Jocassee Dam for this 'sunny day' failure was a piping-related failure through the West Abutment of the dam. This was derived, in part, based on their review of the Federal Energy Regulatory Commission (FERC) Potential Failure Modes Analysis (PFMA) completed in 2004 and 2009. As such, the inundation analyses focused on the initiation of the Jocassee breach caused by a piping type failure. Based on this, the elevation of the initial piping became an important parameter that was considered.

- Turbine and Spillway Gate Operations at Jocassee:

  The HEC-RAS runs do not include any Jocassee Turbine or Spillway Gate operation for the purpose of lowering the Jocassee lake level by passing water prior to or during the event. The Jocassee Spillway Gates are not used at any time during this dam failure and flood simulation. All four of the Jocassee Turbines are operating in the model prior to and during the dam breach simulation. They are passing a cumulative total flow of 26,600 cubic foot / second (cfs) which is being used to numerically stabilize this unsteady flow model – a typical HEC-RAS modeling convention. However, to offset this flow, an inflow hydrograph is being used of equivalent magnitude to 'feed' Lake Jocassee so that no net change in volume or water level occurs at Lake Jocassee. Therefore, the turbines are passing flow – but not to manage or influence the flood levels, but to stabilize the unsteady flow model.

- Reservoir Elevation: 1110 ft. msl (Full Pond)

  The first HEC-RAS run duplicated the FERC 1992 Emergency Action Plan (EAP) DAMBRK analysis, and used a reservoir elevation of 1108 feet Mean Sea Level (msl). The remaining 100 HEC-RAS runs used a Jocassee reservoir elevation of 1110 feet msl (full reservoir - "sunny day" condition).

  The FERC operating range of the reservoir is between 1080 and 1110 feet msl. Available historical information for Lake Jocassee indicates an average reservoir level of approximately 1102 feet msl from 1974 through 2008. If the level of the reservoir were to reach levels above 1108 feet msl, the Duke Energy Hydro Department would take steps to prevent the level rising above 1110 feet msl. Due to the degree to which the Jocassee reservoir elevation is controlled, the full pond reservoir elevation of 1110 feet msl was conservatively adopted for this parameter.

- Bottom Breach Elevation: 800 ft. msl

  The general shape of a postulated breach is trapezoidal. The trapezoid is defined by the base width, the height, and the inclination of the sides. Alternatively, the average breach width may be substituted for the base width. The height of the trapezoid is defined as the difference in the crest of the dam and the bottom breach elevation.

  Available technical papers do address bottom breach elevation. Mtalto and Lia[1] note that the final breach depth is approximately 70% of the dam height for tests conducted for non-cohesive embankments. The height of Jocassee from the crest to the foundation is 375 feet (1125 feet minus 750 feet). Taking 70% of the height and subtracting from the crest elevation equals 862.5 feet msl. A bottom elevation of 800 feet msl would then be conservative based on this reference. Consideration was given to the material of the dam that would 'settle out' of the inundation flow and thus limiting the breach bottom elevation as shown in model tests by Mtalto and Lia. The extent to which the material would 'settle out' out was difficult to quantify because the majority of Jocassee is composed of random rock, and the degree to which the material would 'settle out' is related to the mass of the material, which would also be random. Thus the breach bottom elevation was conservatively determined to be 800 feet msl. To compensate for material 'settle out' and in turn affecting the flow, Manning's *n* was assigned a higher value immediately downstream of Jocassee in the HEC-RAS input.

- Bottom Breach Width: Case 1-250 ft., Case 2-425 ft., Case 3-600 ft.

  The components that define the breach size are the bottom breach width (or alternatively the average breach width), the bottom breach elevation and the breach side slopes. The

[1] Mtalto, Malisa J, University of Dar Es Salaam- Tanzania; Lia, Leif, Norwegian University of Science and Technology, "Physical Hydraulic Modeling and Non-Cohesive Homogeneous Embankment Exposed to Through and Over Flows", Copyright PennWell Corp., 2009 Water Power Conference.

justification for the bottom breach elevation was addressed above. The breach size is used as an input into HEC-RAS. It should be noted that the breach size is estimated based on historic dam failure data for the final or ultimate breach dimensions, which are the result of passing the complete breach hydrograph through the breached section. Chauhan, et al.[2] noted, "Observations of actual dam failures show that breaches continue to grow during the falling limb of the hydrograph. Thus, the peak breach flow rate would not be expected to occur when the breach size is at its maximum." HEC-RAS grows the breach to its final shape over the breach formation time and stops growing after that time, irrespective of the decreasing portion of the breach hydrograph still passing through the breach. Chauhan, et al. continue "Therefore, it is commonly understood that the use of breach parameters obtained from empirical approaches will lead to significant overestimates of peak breach flow rates, at least for dams with large reservoirs," such as the case for Jocassee. "This bias is expected to occur for all empirical breach parameter estimation approaches that use the final breach width and final breach formation time."

Thus, a balanced approach must be used that evaluates the peak break flow rates versus the postulated breach size. Froehlich[3] provides an empirical approach for estimating the average breach width and the peak flow rate. The estimated average breach width can then be used to determine the postulated breach size. The postulated breach size can be used as input to HEC-RAS and a peak flow rate calculated. By comparing the HEC-RAS calculated peak flow rate with the estimated peak flow rate from Froehlich, an adjusted breach size can be determined but was not done for these assessments allowing for additional conservatism.

The average breach width estimated by Froehlich is calculated by the following formula:

$$B = 0.27\ k_o\ V_w^{.33} \qquad (1)$$

Where k_o is a factor for the method of failure, and equals 1.0 for piping failures.
V_w is the volume of water in the reservoir above the breach bottom at the time of failure, in cubic meters.

The volume of water in the Jocassee reservoir at full pond conditions (1110 feet msl) is 1,160,296.0 acre-feet. The total volume is used to compute the maximum possible breach width. This converts to a value of 1,431,203,905 cubic meters. Using this volume, B = 283.6 meters, or approximately 930 feet. Using a 0.7 to 1 side slope as recommended by Froehlich for non overtopping failure modes, the bottom breach width equates to approximately 702.5 feet.

[2] Chauhan, Sanjay S. (Research Assistant Professor of Civil and Environmental Engineering, Utah State University), Bowles, David S. (Professor of Civil and Environmental Engineering, Utah State University), and Anderson, Loren R. (Professor and Head of Civil and Environmental Engineering, Utah State University) "Do Current Breach Parameter Estimation Techniques Provide Reasonable Estimates For Use In Breach Modeling?"

[3] Froehlich, David C, "Embankment Dam Breach Parameters and Their Uncertainties" Journal of Hydraulics Engineering, Vol. 134, No. 12, December 1, 2008, American Society of Civil Engineering.

A review of the actual dimensions of the Jocassee dam indicate that the bottom breach width calculated via Froehlich is not possible at the bottom breach elevation of 800 feet msl due to physical limitations. The maximum possible width at elevation 800 feet msl is approximately 600 feet (prepared foundation).

The 1992 EAP DAMBRK analysis used a bottom breach width of 250 feet. For the HEC-RAS sensitivity analysis the lower bound width was set to this value and the upper bound set at 650 feet. The upper bound value of 650 feet was used only when the bottom breach elevation was selected as 850 feet msl. A value of 625 feet was used only when the bottom breach elevation was selected as 825 feet msl. The variation in this bottom breach width parameter ranges from 250 feet to 650 feet with specified values of 250 feet, 425 feet, 500 feet, 600 feet, 625 feet, and 650 feet. See RAI Question 6 to evaluate sensitivity of this varying breach size with regard to flood heights at the Oconee site.

Froehlich[4] also predicts the peak flow through the breach by the following equation:

$$Q_p = .607\ V_w^{.295} H_w^{1.24} \qquad (2)$$

Where V_w is the volume of water in the reservoir above the breach bottom at the time of failure in cubic meters and H_w is the height of the water above the breach in meters.

Using the volume calculated earlier (1,160,296.0 acre-feet) and noting that the height of the water above the breach is approximately 94.5 meters, the peak flow is calculated as 85,819.8 cubic meters / second or 3,030,699 cubic feet / second (cfs).

Three Jocassee breach sizes were evaluated with HEC-RAS to determine input for the 2-D models. These three cases are as follows:

Case 1

- Reservoir Elevation: 1110 ft. msl
- Bottom Breach Elevation: 800 ft. msl
- Bottom Breach Width: 250 ft.
- Time to Failure: 2.6 hrs.
- Side Slopes, both sides of breach: (1:1)
- Piping Elevation: 1020 ft. msl
- Failure Progression: Sine Wave

Case 2

- Reservoir Elevation: 1110 ft. msl
- Bottom Breach Elevation: 800 ft. msl
- Bottom Breach Width: 425 ft.
- Side Slopes, West Slope: (1.55:1); East Slope: (0.7:1)
- Time to Failure: 2.8 hrs.
- Piping Elevation: 1020 ft. msl
- Failure Progression: Sine Wave

[4] Froehlich, David C, 1995b. "Peak Outflow from Breach Embankment Dam", Journal of Water Resources Planning and Management, Vol. 121, No. 1, p. 90-97

Case 3

- Reservoir Elevation: 1110 ft. msl
- Bottom Breach Elevation: 800 ft. msl
- Bottom Breach Width: 600 ft.
- Side Slopes, West Slope: (1.55:1); East Slope: (0.7:1)
- Time to Failure: 3.0 hrs.
- Piping Elevation: 1020 ft. msl
- Failure Progression: Sine Wave

The HEC-RAS results for these three cases indicated peak flow rates through the Jocassee breach as follows:
Case 1 – 4,760,000 cfs
Case 2 – 5,440,000 cfs
Case 3 – 6,300,000 cfs

A comparison of the HEC-RAS calculated peak breach flow rates with the Froehlich estimated peak flow rate indicates that HEC-RAS, with the given input breach sizes, over predicts the peak flow rates for all three cases and by as much as 200% for Case 3. These results confirm the expected outcome with regard to overestimated peak flow determinations as noted by our research presented earlier in this section. Since the HEC-RAS peak flow rate for the Case 2 breach size provides a substantial margin (almost an 80% increase over the Froehlich methodology) with regard to Froehlich's estimated peak flow, Case 2 breach size was adopted as the conservative case.

- Side Slopes: West Slope: (1.55:1); East Slope: (0.7:1)

The side slopes help define the breach trapezoidal area. Froehlich recommends a 0.7 horizontal to 1.0 vertical (0.7:1) side slopes for the final breach shape for non-overtopping failure modes. A piping initiated failure was the failure mode considered for the 'sunny day' failure of Jocassee Dam. It was also recognized that in order to capture the location of the West Abutment seepage, the West side slope must be raked more to the horizontal. To capture the West side seepage, the slope was raked to 1.55:1. This is essentially the slope of the natural ground of the west side of the dam.

The East Abutment seepage is significantly less in magnitude than the West Abutment seepage, therefore, a modification of the East side slope was not considered. The east slope was adopted to be 0.7:1 in accordance with Froehlich. The combination of side slopes for the East and West sides of the postulated breach was also varied to capture the difference in seepage between the two abutments. These final side slopes result in a larger projected breach area than if the Froehlich recommended side slopes were used for both side slopes. Consequently, this side slope combination provides another degree of conservatism for the final Cases 1, 2, and 3.

- Time to Failure: Case 1= 2.6 hours, Case 2=2.8 hours, Case 3=3.0 hours

Time to failure is defined as the time between the onset of piping to the attainment of the full breach size. Froehlich's methodology estimated a time to failure of 2.2 hours for Jocassee

Dam. Walder and O'Connor[5] estimated a time to failure of 3.04 hours. A value of 2.6 hours was used for Case 1 (the smallest breach). This was considered an average of the Froehlich estimated time to failure and the Walder and O'Connor estimated time to failure. A value of 3.0 hours was used for Case 3. For Case 2, a value of 2.8 hours was used. The justification for this value recognizes that breach size and time to failure are related. As such, a larger breach would have a longer time to failure versus a smaller breach. The value of 2.8 hours was adopted, since the Case 2 breach size is larger than the Case 1 breach size, and thus the time to failure for Case 2 is slightly larger than the Case 1 time to failure. The sensitivity of time to failure for Jocassee Dam as it relates to determining flood heights at the Oconee site is discussed more in RAI Question 6.

- Initial Piping Elevation: 1020 ft. msl

  During the literature search, no technical papers were found that predicted the probable piping elevation. The HEC-RAS software tool allows the input of this parameter, so variation in this parameter was pursued. The 1992 EAP DAMBRK analysis assumed a piping elevation of 940 feet msl. As noted before, the sensitivity study attempted to capture the potential failure mode associated with seepage through the West Abutment. The West Abutment seepage is generally located at elevation 1020 feet msl. Thus, the initial piping elevation was varied at the specific values of 940 feet and 1020 feet msl. For Cases 1, 2, and 3, the piping elevation of 1020 feet msl was adopted to coincide with the elevation of the actual seepage path on the West Abutment.

- Failure Progression: Sine Wave

  Available technical papers do not address failure progression. The sine wave progression was adopted due to the more reasonable representation of the material loss through the breach provided by the sine wave progression versus the linear progression. The sine wave progression predicts a gradual increase in material loss as the breach begins to develop with a more pronounced increase as the breach grows before tapering off to its maximum size. The sine wave method of progression is more representative of the postulated failure than the linear method considering a piping breach would result in a rapid collapse of the crest of the Jocassee dam as the pipe through the dam develops.

[5] Walder, Joseph S., Cascades Volcano Observatory, United States Geological Survey, Vancouver, Washington; and O'Connor, Jim E., United States Geological Survey, Portland, Oregon. "Methods for Predicting Peak Discharge of Floods Caused by Failure of Natural and Constructed Earthen Dams", Water Resources Research, Vol. 33, No. 10, Pages 2337-2348, October 1997.

Keowee Parameters[6]

Early analysis indicated that the Keowee Main Dam, the West Saddle Dam, and the Oconee Intake Dike and Little River Dam structures would be overtopped following a postulated breach of the Jocassee Dam. Therefore, it is important to understand whether these structures fail, how they fail, and the timing of their failure. The Little River Dam is located several miles from the Oconee site and its failure does not directly affect the Oconee site. However, if the Little River Dam fails, it could help relieve water away from the flooded Lake Keowee. Therefore, it is important to understand whether the structures fail, to what degree they fail, and the timing of the failure.

- Reservoir Elevation: 800 ft. msl (Full Pond)

  Due to the rigorous level of control to which the Keowee reservoir elevation is controlled, the full pond reservoir elevation, 800 feet msl was adopted as the initial reservoir elevation for all runs in the sensitivity analysis (with the exception of the first run used for comparison to the 1992 EAP work). The actual average reservoir level for Keowee is approximately 796.7 feet msl based on historic information available from 1971 through 2008.

Keowee Main Dam

- Bottom Breach Elevation: 670 ft. msl

  A value of 670 feet msl was selected for this parameter.

  The Keowee Main Dam is assumed to fail as the inundation flow from the postulated failure of Jocassee overtops the structure. Bottom breach elevations of 670 feet msl and 700 feet msl were explored in the sensitivity analysis. The 670 foot value corresponds to an approximate base elevation of the dam where a reasonable bottom breach width can be achieved due to the shape of the prepared bed. (The actual prepared bed elevation at the Keowee main dam is approximately 650 feet msl).

- Bottom Breach Width: 500 ft.

  Using Froehlich methodology, the estimated bottom breach width is 1028 feet, well beyond the actual width of the Keowee Main Dam near foundation. The actual bottom width of 500 feet between the abutments was the value used for this parameter.

  Froehlich methodology provides an empirical approach for estimating the average breach width. The estimated average breach, along with the side slopes and breach height, can then be used to determine the bottom breach width. Froehlich estimated an average breach width of 1173 feet. The physical size of the dam limits the maximum average breach width to approximately 897 feet. In addition, the same drawing indicates that the maximum bottom breach width is approximately 500 feet.

[6] All of the Keowee impoundment structures, e.g. the Main Dam, the West Saddle Dam, the Oconee Intake Dike, and the Little River Dam are of similar construction and material.

Bottom breach widths of 500 and 650 feet respectively were explored in the sensitivity analysis. The 500 foot value was used in combination with the breach bottom elevations of both 670 and 700 feet msl. However, the 650 foot width value was only used in combination with the breach bottom elevation of 700 feet msl due to the physical constraints of the Main Dam.

- Side Slopes: (1:1)

The value used for side slopes was based on Froehlich methodology (1:1) for an overtopping failure mode.

Froehlich estimated a 1 horizontal to 1 vertical (1:1) side slope for the final breach shape for overtopping failure modes. Flatter side slopes were also explored in the sensitivity analysis. The use of the flatter slopes allowed postulation of larger projected area breaches of the Keowee Main Dam. In some cases unequal side slopes were explored. This again allowed postulation of larger projected area breaches of the Keowee Main Dam.

For the cases where the side slopes were equal on both sides of the postulated breach, the slope values of 1:1 and 1.5:1 were used. For the cases where the side slopes were unequal, the slope combinations of 3.45:1 (West) and 2.03 (East) were used.

The slope value of 1:1 is fully justified by the Froehlich estimated side slope given above. The larger side slopes and hence larger breach sizes were explored to understand the effect of allowing more flow through the Keowee Main Dam breach and its possible effect on the level of water at the Oconee site.

- Time to Failure: 2.8 hours

The time to failure parameter value adopted for the final cases was 2.8 hours.

This parameter value is based on a proportional factor applied to the Froehlich methodology estimated time to failure value of 5 hours. This approach recognizes that time to failure and breach size parameters are related. The proportional factor is equal to the average breach width for a 1:1 side slope breach, with the aforementioned bottom breach width of 500 feet with a height of 145 feet (815 feet minus 670 feet) divided by the average breach width estimated by Froehlich. The average breach width calculated based on 1:1 side slopes and the above noted parameters is 645 feet. Using the Froehlich methodology estimated an average breach width of 1173 feet. Thus, the proportional factor is 0.55 (or 645/1173). Multiplying the proportional factor by the Froehlich methodology estimated time to failure of 5 hours yields 2.8 hours. The main dam's time to failure is used as a base for determining the time to failure of the remaining earthen structures surrounding the Keowee reservoir due to the similarity of the material used in the dams and the common construction practices.

Time to failures values of 2 hours, 2.4 hours, 2.8 hours, and 4 hours were explored in the sensitivity analysis. This sensitivity can be better understood by seeing the response to RAI Question 6.

- Overtopping Trigger: 817 ft. msl (2 ft. above crest)

  An overtopping trigger of 2 feet was selected based on several references and the conclusions reached by the experts after completion of the initial HEC-RAS runs which included both 0.5 and 2 foot overtopping triggers.

  The HEC-RAS work supported calculated velocities at the crest and at the downstream face of the dam 2 to 4 times the accepted velocity exposure limits for steep grassed slopes. Also, the experts confirmed the lack of sensitivity regarding this parameter as seen in the multiple HEC-RAS runs. The lack of sensitivity for this value with regard to resulting flood heights at the Oconee site is primarily due to the rapid rate of rise of the overtopping waters.

- Failure Progression: Sine Wave

  The Sine Wave failure progression was adopted for all embankment structures; see justification given under Jocassee parameters.

West Saddle Dam

Cases were explored in the sensitivity analysis assuming the West Saddle Dam failed and did not fail after being overtopped. Both cases were considered to gain a better understanding of whether failing the West Saddle Dam would affect the water level at the Oconee site, either from a backwater affect in the Keowee tailrace if the West Saddle Dam failed or by increased overtopping level at the Oconee Intake Dike if the West Saddle Dam did not fail.

- Bottom Breach Elevation: 795 ft. msl

  The adopted bottom breach elevation is based on the actual toe elevation (795 feet msl) of the majority of the dam. The sensitivity of this individual parameter for the West Saddle Dam was not pursued in this work.

- Bottom Breach Width: 1680 ft.

  The adopted bottom breach width of 1680 feet is based on the assumption that ~80% of the west saddle dam fails following overtopping. This exceeds the average breach width value based on the Froehlich method, 1173 feet. The sensitivity of this individual parameter for the West Saddle Dam was not pursued in this work.

- Side Slopes: 0:1

  Given the relatively short size of the structure (20 feet high), the breach side slopes are assumed to be vertical. Side slopes for a structure of this height is not a sensitive parameter. The sensitivity of this individual parameter for the West Saddle Dam was not pursued in this work.

- Time to Failure: 0.5 hours

  The adopted time to failure of 0.5 hours is based on the assumption that the dam will rapidly fail once overtopped due to rapidly increasing and high velocity flow over the downstream face of the dam. Also, the smaller size (height) of the West Saddle Dam would support it having a failure time smaller than the other impoundment structures. The sensitivity of this individual parameter for the West Saddle Dam was not pursued in this work.

- Overtopping Trigger: 817 ft. msl (2 ft. above crest)

  The overtopping trigger is discussed under Keowee Main Dam. The sensitivity of this individual parameter for the West Saddle Dam was not pursued in this work.

- Failure Progression: Sine Wave

  The Sine Wave failure progression was adopted for all embankment structures; see justification given under Jocassee parameters. The sensitivity of this individual parameter for the West Saddle Dam was not pursued in this work.

Oconee Intake Dike

In those cases where the Oconee Intake Dike was overtopped, a breach was assumed to develop on the portion of the dike facing east. (See other RAIs in this response for the discussion regarding the breaching of the Oconee Intake Canal Dike.) The assumption that the dike would fail when overtopped is based on historical data that indicates that earthen dams fail when significantly overtopped.

- Bottom Breach Elevation: 715.5 ft. msl

  The adopted bottom breach elevation is based on the actual toe elevation (approximately 715.5 feet msl) of the portion of the intake dike facing east. (See other RAIs in this response for the discussion regarding the breaching of the Oconee Intake Canal Dike.) The sensitivity of this individual parameter for the Oconee Intake Canal Dike was not pursued in this work.

- Bottom Breach Width: 200 ft.

  Based on the Froehlich methodology, the estimated bottom breach width would be 1173 feet, well beyond the actual width of the Oconee Intake Dike. Due to physical constraints, the Bottom Breach width parameter was set to 200 feet. The sensitivity of this individual parameter for the Oconee Intake Canal Dike was not pursued in this work.

- Side Slopes: (1:1)

  The side slopes value was based on Froehlich methodology (1:1) for an overtopping failure mode. The sensitivity of this individual parameter for the Oconee Intake Canal Dike was not pursued in this work.

- Time to Failure: 0.9 hours

  The adopted time to failure was based on a proportional factor applied to the projected area of the Keowee Main Dam breach. The projected area of the Keowee Main Dam breach referenced above is 93,525 square feet (sq. ft.). The projected area of the adopted Oconee

Intake Dike breach, described above, is 29,800 sq. ft. The proportional factor is then 29,800/93,525 = 0.3186. Recognizing that the time to failure is proportional to the breach size and the adopted time to failure of the Keowee Main Dam is 2.8 hours, the time to failure of the Oconee Intake Dike is then 0.3186 x 2.8 hours = 0.9 hours.

- Overtopping Trigger: 817 ft. msl (2 ft. above crest)

  The overtopping trigger is discussed under Keowee Main Dam. The sensitivity of this individual parameter for the Oconee Intake Canal Dike was not pursued in this work.

- Failure Progression: Sine Wave

  The Sine Wave failure progression was adopted for all embankment structures; see justification given under Jocassee parameters. The sensitivity of this individual parameter for the Oconee Intake Canal Dike was not pursued in this work.

Little River Dam

In those cases where the Little River Dam was overtopped, a breach was assumed to develop. The assumption that the dam would fail when overtopped is based on historical data that indicates that earthen dams fail when significantly overtopped.

- Bottom Breach Elevation: 670 ft. msl

  The adopted bottom breach elevation was taken as 670 feet msl, consistent with the Main Keowee Dam and consistent with physical valley cross section. The sensitivity of this individual parameter for the Little River Dam was not pursued in this work.

- Bottom Breach Width: 290 ft.

  The Froehlich methodology estimated an average breach width of 1173 feet, far beyond the actual width of the Little River dam. Due to physical constraints, the adopted bottom breach width is 290 feet. The sensitivity of this individual parameter for the Little River Dam was not pursued in this work.

- Side Slopes: (1:1)

  The side slopes value was based on Froehlich methodology (1:1) for an earthen dam. The sensitivity of this individual parameter for the Little River Dam was not pursued in this work.

- Time to Failure: 1.9 hours

  The adopted time to failure is based on a proportional factor determined from the projected area of the Keowee Main Dam breach. The projected area of the Keowee Main Dam breach referenced above is 93,525 sq. ft. The projected area of the adopted Little River Dam breach, described above, is 63,075 sq. ft. The proportional factor is then 63,075/93,525 = 0.6744. Recognizing that the time to failure is proportional to the breach size, and the adopted time to failure of the Keowee Main Dam is 2.8 hours, the time to failure of the Oconee Intake Dike is then 0.6744 x 2.8 hours = 1.9 hours.

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- Overtopping Trigger: 817 ft. msl (2 ft. above crest)

  The overtopping trigger is discussed under Keowee Main Dam. The sensitivity of this individual parameter for the Little River Dam was not pursued in this work.

- Failure Progression: Sine Wave

  The Sine Wave failure progression was adopted for all embankment structures, see justification given under Jocassee parameters. The sensitivity of this individual parameter for the Little River Dam was not pursued in this work.

2. **Question:**

Justify the use of the different Manning *n* values for the following areas: Little River Basin below the channel, the Keowee River below the Jocassee tailrace, and in the Keowee tailrace extension to the road bridge. Specifically, describe how the *n* values selected represent a conservative value.

Response:

The original March 2009 HEC-RAS Model (Run # 1) generally utilized a Manning's *n* value, or roughness coefficient, of 0.035 for the main channel and 0.08 for overbank areas throughout the model. These Manning's *n* values were used in tributaries, reservoirs, and river sections. These values were used exclusively for HEC-RAS sensitivity runs 1 through 63.

Additional research indicated that the Manning's *n* value in most streams decreases with an increase in stage and discharge, and generally approaches 0.02 in deep reservoirs. Therefore, later model runs were performed with Manning's *n* values of 0.02 and 0.025 for the main reservoir channels for Jocassee, Keowee, and Hartwell reservoirs. Each individual river reach and cross section in the HEC-RAS Model was reviewed to generally discern water depths in excess of 60 ft. The 60 foot threshold was arbitrarily chosen to discern reservoir and riverine cross sections. Manning's *n* values were revised from 0.035 to 0.02 (early runs) and 0.025 (later runs) using the 60 foot threshold in the main channel. Reservoir tributary *n* values remained at 0.035. A Manning's *n* value of 0.08 was held constant for all overbank areas.

In addition, the Manning's *n* values were revised from 0.035 to 0.07 in the respective tailrace reaches below Jocassee Dam, Keowee Dam, ONS Intake Canal Dike, and Little River Dam to account for roughness associated with displaced dam breach material (suspended material and bed load). It is recognized that the Jocassee Dam (rock fill) material is different from the Keowee Reservoir dams (compacted soil). Both suspended material and bed load consume energy and cause an apparent increase in Manning's *n*. No attempt was made to evaluate Manning's *n* values for the respective dam locations and as such, a conservative value of 0.07 was assigned to the tailrace reach below each dam. A higher tailrace n value is assumed to result in a more conservative tailrace elevation profile below each breached structure and is believed to be applicable in this modeling effort. Based on the differences in the construction material of Jocassee verses the Keowee impoundment structures, it would be expected that the channel below the Jocassee breach would be rougher, with a corresponding higher Manning's n than that expected below the respective Keowee impoundment structures breached. However, the same n value is being used for the earthen dams (Lake Keowee impoundment structures) as for the Jocassee Dam (rockfill structure), thus a higher degree of conservatism is expected in determining the Keowee tailrace water surface elevations and the associated potential flooding of the Oconee site.

The affected reach lengths below each dam were limited to the base width dimension of the respective dams, followed by a second base width dimension (the width of the dam from the upstream toe to the downstream toe) to allow transition from 0.07 to 0.02. For example, the base width of Jocassee Dam is approximately 1300 feet at elevation 800 feet msl. Therefore, the Jocassee Dam tailrace would have a Manning's *n* value of 0.07 for a distance of 1300 feet below the downstream toe of the dam and an additional 1300 feet to transition from 0.07 to 0.02. The Manning's *n* transition was assumed to be linear.
Considering the Manning's *n* value transitioned during the various assessments using the

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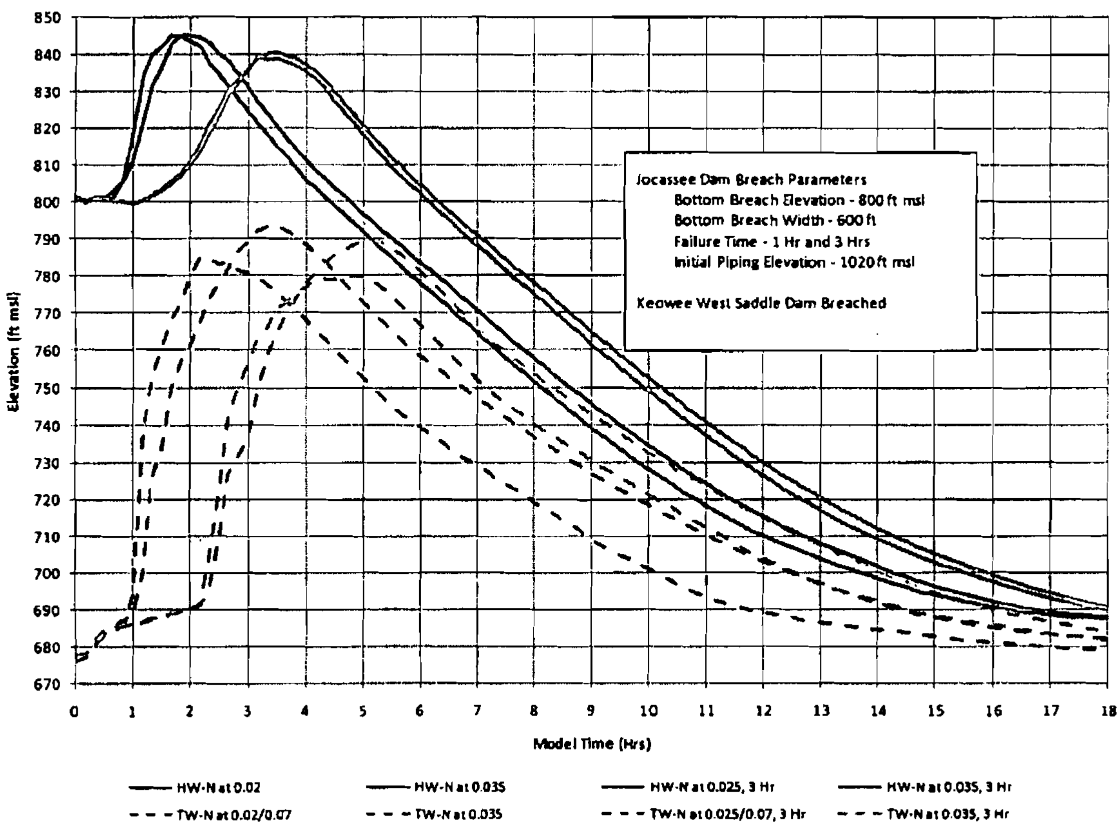
HEC-RAS model, a review of the sensitivity was conducted. The Manning's *n* value transitioned as shown below for the main channel for both the Jocassee tailrace (Lake Keowee) and the Keowee tailrace (Lake Hartwell).
Model Runs 1–63: 0.035
Model Runs 64–76: 0.02
Model Runs 77–101: 0.025

The Figure below provides the relative stage hydrograph comparisons at Keowee Dam for Manning's *n* values of 0.02, 0.025, and 0.035. The headwater, or HW, shall be considered the water located upstream of the Keowee Dam. The tailwater, or TW, shall be considered the water located downstream of the Keowee Dam.

MANNING'S *N* VALUE SENSITIVITY



Manning's *n* value has a direct impact on the stage hydrograph timing of the headwater and tailwater events. Lower n values result in a stage hydrograph shifts so as to reduce time to peak water surface elevations for the tailwater area below Keowee Dam. The variation of n values had minimal impact on the headwater side of the Keowee Dam. The transitioning in *n* values also revealed under similar breach parameter conditions, the tailwater stage hydrograph peak elevations tend to increase with a Manning's *n* value of 0.035 than with the n value of 0.07 with transitioning to 0.02/0.025.

The Keowee tailrace backwater is more susceptible to the downstream conditions (Manning's *n* value of 0.035 versus 0.02/0.025) due to the overall backwater effect of the Keowee River/Lake Hartwell headwaters.

In summary, the Manning's *n* value selected and used for Cases 1, 2 and 3 (also referenced as runs 101, 100, and 99) should produce a more conservative water surface elevation in the Keowee tailrace area and the Oconee site.

3. Question:

Justify the armoring of the intake canal in your model. Specifically, describe what effects the armoring of the intake canal has in directing the flow of water during a potential breach and in the location of the breach. Also identify areas of the plant where water is likely to be higher without the armoring, as well as with the armoring of the intake canal. Describe how these assumptions represent a conservative value.

Response:

The 1-D (or HEC-RAS) and 2-D models have assumed the breach of the Oconee Intake Canal Dike occurs on the east side of the embankment structure and is considered reasonable for the model work for the following reasons:

1. The momentum and flow direction of the water through the canal is in the eastern direction due to the training the water receives as it flows through the long canal. Water will fill the northern end of the canal after it has been redirected and slowed due to the Intake Structure itself in the canal and the eastern side of the dike. Water filling the northern end must flow through the narrow area between the eastern edge of the Intake Structure and the dike itself in order to fill this area. Therefore, it is expected the flow tendency will be to the east, and once the canal is full to the crest, it was assumed it would overtop on the eastern side first (due to momentum), hence the breach forming on the eastern side and relieving the canal flows.
2. The Intake Structure for the Condenser Circulating Water (CCW) system is a large reinforced concrete structure positioned east-west within the northern end of the canal. This structure will have an influence on the flow regime and velocities of any flow headed to the north due to the canal being filled during the flood event. The top of the reinforced concrete Intake Structure is at 810 feet msl as compared to the crest of the Intake Structure Dike at 815 feet msl. However, a total of 12 large pump motors heavily anchored to the structure protrude approximately 14 feet above the top of the Intake Structure. Therefore, it was concluded the reinforced concrete structure, and the protruding pump motors would have significant influence on slowing velocities to the north, and encouraging the flood flows to follow their natural tendencies and flow to the east over the eastern portion of the dike to a greater degree than to the northern side of the dike.
3. The eastern side is where the Oconee Intake Canal Dike has the tallest cross-section of embankment allowing the larger breach to be assumed. The northern side of the dike is much shorter from the crest to the toe, approximately 19 feet (using a toe elevation of the yard, or 796 feet msl), verses the eastern side where the distance from the crest to the toe is approximately 95 feet (using an approximate toe elevation of 720.0 feet msl). The velocities attained by the crest overflows would be significantly greater on the eastern side than the northern side, and therefore more damaging to the embankment of the dike on the eastern side.
4. The flow volume possible out of the canal on the eastern side breach is significantly greater than any breach of the northern side of the dike. The canal dike can erode to the bottom of the canal, or elevation 760.0 feet msl on the eastern side – but this is not possible on the northern side due to the yard having an elevation of 796 feet (that is approximately 36 feet higher than the bottom of the Intake Canal). Therefore, it was concluded the failure to the east would be more conservative than a failure to the north since it would produce larger flows to the tailrace area.

In summary, due to the direction and momentum of the water, the reinforced concrete CCW Intake Structure at the northern end of the canal, the more damaging velocities attained on the eastern side of the dike, and the greater breach size achievable on the eastern side of the dike, it was assumed the eastern side of the Intake Canal Dike was most likely to fail as well as produce the greatest consequences for the tailrace area. Also, once the eastern side of the Intake Canal Dike starts failing, the likelihood of a large failure to the northern side of the canal dike is further reduced due to the flood waters being relieved from the intake canal through the eastern breach and the short duration of overtopping on the northern side. Modeling the breach on the eastern side of the dike allows the larger flows out of the Intake Canal and into the Keowee tailrace. Therefore, the currently modeled breach contributes to generating higher water levels in the tailrace.

Since the breach was assumed to occur only on the eastern portion of the dike (no failure on the northern side of the dike), it was implied we had assumed the northern side was armored since no failure occurred in this location. The actual armoring of this side would further reinforce the confidence of our active model. Currently, no armoring, or additional protection, exists for the northern side of the Intake Canal Dike. Although armoring would assure the integrity of the dike during this potential overtopping, the lack of armoring would not necessarily lead to complete destruction of the dike on this northern end.

Any future armoring of the northern side of the dike would not influence over-the-crest flows towards the Standby Shutdown Facility (SSF) per the model as it is assembled today. Also, armoring the northern side of the dike would not influence the postulated breach size or location on the eastern side of the dike. Instead, the armoring would further assure no failure, or breach, would occur on the northern side of the dike.

If a failure were postulated to occur on the northern side of the dike, the resulting flood heights in the general area of the SSF are estimated to be slightly higher than the current heights per our assessments. This is strictly based on how the model would be expected to respond. A breach on the northern side of the dike is not currently modeled in our failure scenarios; therefore we do not have calculated results to report for this case. Below are considerations to support an estimate of consequences for a breach on the northern side of the dike:

- The peak water elevation should not be any lower in the general area of the SSF, but should be slightly higher; the impact to the station is essentially the same.
- The peak water elevation is already high, but very short duration (narrow peak); a breach on the northern side of the canal would most likely lengthen the duration (or broaden the peak).
- Due to the CCW Intake Structure's robustness and location, any increased flow to a failure on the northern side of the dike would require flows around the eastern edge of the Intake Structure, thus encouraging a failure of the eastern portion of the dike as well, thus reducing the total cumulative flow directed to the northern breach.
- The CCW reinforced concrete Intake Structure would need to be modeled as a barrier to the possible flow out of the Intake Canal if a breach on the northern side of the Intake Canal Dike were to be incorporated into the 2-D model.

In conclusion, the armoring of the northern side of the Intake Canal Dike was assumed to support modeling a larger breach in the eastern side that contributes to generating higher water levels in the tailrace. Should a breach be assumed in the northern side, slightly higher water levels would be postulated at the SSF, but impact to the station would remain essentially the same.

4. Question:

The 2-dimensional (2-D) model shows a second surge of water at the Oconee site due to a backup of water from the Keowee tailrace. Describe the effect of the overall water level at the Oconee site, following a faster breach time of the Keowee Dam.

Response: (Interim)

The 2-Dimensional (2-D) models, associated with Cases 1, 2, and 3 as presented to the NRC on October 28, 2009, revealed two distinct peak water heights occurring in the ONS yard and around the SSF. The 1st peak is due primarily to water flowing over the Intake Canal dike crest and collecting in and around the SSF due to current plant structures controlling the draining capability of the yard. As a result, water is temporarily detained (similar to a detention pond) in and around the SSF while the flood waters are both collected and released by the Oconee site.

The 2nd peak water height in the 2-D model is due to the flood waters collecting in the Keowee Tailrace area and flooding the site. Essentially the Keowee tailrace is the ultimate collection point for waters from the Keowee Main Dam, Keowee West Saddle Dam, the swale at the World of Energy, and the Oconee Intake Canal Dike. Based on these cases, this 2nd peak is never higher than the 1st peak for all cases.

The 1-dimensional (1-D) HEC-RAS runs (reference RAI Question 6) did calculate a tailwater elevation below the Keowee Dam. The tailwater value from HEC-RAS represents essentially the same data point in the flood analysis as the 2nd peak does in the 2-D work (This would be based on the assumption that the water surface is level from the Keowee tailrace to the SSF area – this may or may not be the condition of the water surface at all points in time during the event being modeled.) Although the values are different, they can be compared in a general sense to represent the same point of reference between the two models. The chart below summarizes these results. (Note: The 1-D model does not determine a water height around the SSF due to flow over the Oconee Intake Canal Dike since this software tool is being used primarily to determine the global characteristics of the event – not the local results such as water height around the SSF.)

	Case 1		Case 2		Case 3	
Maximum Occurring Water Heights at SSF:	HEC-RAS	2-D	HEC-RAS	2-D	HEC-RAS	2-D
Tailwater elevation	772.5	See 2nd peak*	776.7	See 2nd peak*	780.0	See 2nd peak*
1st Peak water height	N/A	813.5 ft. msl	N/A	814.5 ft. msl	N/A	815.5 ft. msl
2nd Peak water height	N/A	799.7 ft. msl	N/A	803.2 ft. msl	N/A	807.2 ft. msl

* For purposes of comparison, the 2-D 2nd peak water height shall also be considered the tailwater elevation

The 1st and 2nd peak values at the SSF are only calculated specifically in the 2-D model work – they are not determined in the HEC-RAS runs. Therefore, the impact on the water levels at the Oconee site due to a faster breach time for the Keowee Dam can only be determined with additional 2-D assessments.

Additional 2-D assessments are underway to determine the impact of faster breach times for the Keowee Main Dam as well as other Keowee impoundment structures having an influence on the resulting tailrace elevation below the Keowee Dam. This additional scope of work will provide updated information and may affect the overall water levels at the site. Therefore, the information provided is considered an interim response. A final response to this RAI will be submitted in June 2010.

5. Question:

Organize the final runs such that the set of parameters that provides the highest water level for each point of interest (flood barrier, standby shutdown facility, and other necessary points of personnel ingress, etc.) can be identified and evaluated with reasonable conservatism.

Response:

For this RAI response and based on clarifying discussions with the NRC staff on January 20 and March 3, 2010, 'final runs' shall be considered the 3 final runs presented to the NRC on October 28, 2009, also known as Cases 1, 2, and 3. More specifically, for this RAI response, the 'final HEC-RAS run and 2-D model' used to assess potential site flooding is Case 2.

Presented below are the maximum water levels (or water surface elevations) for the points of interest per Cases 1, 2, and 3. Both HEC-RAS runs and 2-D results are being presented for clarity and completeness. The parameters associated with each case are shown below (consistent between both the HEC-RAS and 2-D models).

Peak Water Elevation Results:

Points of interest:	Elevation/HEC-RAS case	Elevation/2-D case
Peak water height over Keowee Dam	839.4 ft. msl/Case 3	841.6 ft. msl/Case 3
Peak water height through swale	830.8 ft. msl/Case 3	829.0 ft. msl/Case 3
Peak water height over Intake Dike	823.1 ft. msl/Case 3	823.0 ft. msl/Case 3
Peak Tailrace surface elevation	780.0 ft. msl/Case 3	Same as 2nd peak below*
Peak water height at SSF -1st	N/A	815.6 ft. msl/Case 3
Peak water height at SSF – 2nd	N/A	807.5 ft. msl/Case 3

* For purposes of comparison, the 2-D 2nd peak water height shall also be considered the tailwater elevation

Supporting Case Parameters:

Parameter:		Case 1	Case 2	Case 3
Reservoir Elevation		1110 ft. msl	1110 ft. msl	1110 ft. msl
Breach Bottom Elevation		800 ft. msl	800 ft. msl	800 ft. msl
Breach Bottom Width		250 ft.	425 ft.	600 ft.
Side Slopes		1:1 for both	W1.55:1. E 0.7:1	W1.55:1. E 0.7:1
Piping Elevation		1020 ft. msl	1020 ft. msl	1020 ft. msl
Time to Failure		2.6 hrs.	2.8 hrs.	3.0 hrs.
Failure Progression		Sine wave	Sine Wave	Sine Wave

Parameters Common to all three Final Cases:

Keowee Main Dam		800 ft. msl starting elevation
Breach Bottom Elevation		670 ft. msl
Breach Bottom Width		500 ft.
Side Slopes		1:1
Overtopping Trigger		817 ft. msl
Main Dam Failure Time		2.8 hrs.
West Saddle Dam		
Breach Bottom Elevation		795 ft. msl
Breach Bottom Width		1680 ft.
Side Slopes		1:0
Overtopping Trigger		817 ft. msl
WSD Failure Time		0.5 hrs.
ONS Intake Canal Dike		
Breach Bottom Elevation		715.5 ft. msl
Breach Bottom Width		200 ft.
Side Slopes		1:1
Overtopping Trigger		817 ft. msl
OID Failure Time		0.9 hrs.

Little River Dam		
Breach Bottom Elevation		670 ft. msl
Breach Bottom Width		290. ft.
Side Slopes		1:1
Overtopping Trigger		817 ft. msl
LRD Failure Time		1.9 hrs.
Manning's *n* values		
Jocassee Tailrace		0.07
Keowee Reservoir Channel		0.025
Keowee Reservoir Out of Channel		0.08
Keowee Reservoir Tributaries		0.035
Keowee, Intake and Little River Tailraces		0.07
Hartwell Reservoir Channel		0.025
Hartwell Reservoir Out of Channel		0.08

6. **Question:**

Provide the key for your runs associated with the sensitivity analysis for the Hydrologic Engineering Center – River Analysis System (HEC-RAS) model.

Response:

The HEC-RAS runs 1 through 98 shall be considered the 'sensitivity analysis' information for Cases 1, 2, and 3 (HEC-RAS runs 101, 100, and 99 respectively).

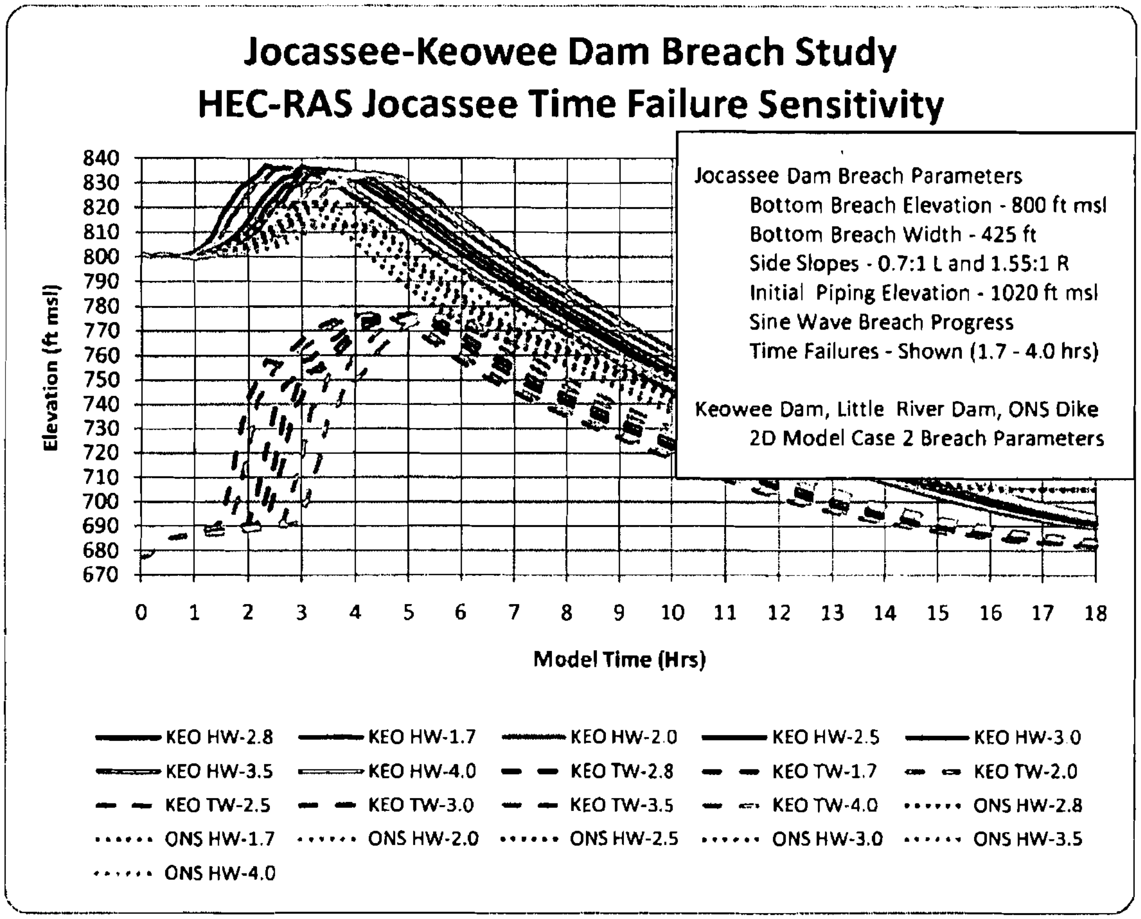
The target being monitored with this sensitivity analysis is the final water heights at and around the Oconee site. In order to do this evaluation, some parameters of the model will need individual sensitivity evaluations to determine their role in the overall outcome, or results. In other words, does or does not the individual parameter play a key role in determining the water height at Oconee. This can be done by varying the parameter and holding all other parameters constant.

The model also has many contributing parameters feeding into one large model input for breach size. All these possible combinations essentially can yield very similar results, or outcomes that actually represent a single large parameter – the breach size. Therefore, if a group of parameters can be collectively represented by a single value, or input parameter, so that the representative outcome reflects the contribution of the individual parameters, then this will be done so as to simplify this assessment. In this case, one parameter reflects the summation of several 'detail' parameters.

If a parameter is held constant, or varied in a known range based on strong technical basis, then this parameter will not be a focus in this sensitivity assessment. The value of this parameter will be presented along with the overall results and its influence will be assessable.

All other pertinent parameters having an influence in the model will be presented for individual evaluations and assessments for each HEC-RAS run.

Starting at the water source for the model, the failure sensitivity components for the Jocassee Dam can be divided into two main categories: the time used for the failure of the dam, and the size of the dam breach itself. Presented below is additional information to show the lack of sensitivity for the resulting water heights around the Oconee site with respect to Failure Time for the Jocassee Dam.



The information above presents the results of this time sensitivity investigation for the failure of Jocassee Dam and the resulting water heights important for the Oconee site. This work was done with regard to Case 2 parameters. For this case, the time of failure for the Jocassee Dam was varied from 1.7 hours to 4 hours holding all other parameters constant in the analysis model. In general, there are no differences with regard to the resulting water heights for the headwater and tailwater for Keowee Dam and the headwater for the Oconee Intake Canal Dike as the Jocassee Dam Failure Time was varied. The arrival time for the peak water elevations varied as the failure time was varied – but the water surface elevations stayed essentially the same. Therefore, the time of failure for a particular size of failure has minimal significance on the overall water heights at ONS. Time of failure will still be presented in the result, but it will be shown not be to be a key player in determining final heights of water for the Oconee site.

In order to simplify all the various combination of parameters that could possibly describe the breach failure size for the Jocassee Dam for this assessment, one value will be calculated to represent these parameter's collective influence. This term will be referred to as the 'Breach Cross Section'. It will represent the cross-sectional area of the final breach size developed using the following parameters: Breach Bottom Elevation, Breach Bottom Width, and Side Slopes.

After reviewing the input data, the HEC-RAS runs and their associated results (water heights) have been presented in this order. Each step increase represents a sub-sort of the data from the step before it.

1. West Saddle Dam not failed and then failed.
2. Order the runs with regard to increasing breach size for Jocassee Dam.
3. Order the runs with regard to decreasing time to failure for Keowee Main Dam.
4. Order the runs with regard to increasing time to failure for Jocassee Dam.

The graph presents the resulting water heights for the Keowee Headwaters, Keowee Tailrace, Oconee Intake Dike Headwaters, and the elevation of the water at the swale near the World of Energy. The sensitivity of the inputs and how they relate to the water heights can be seen.

The data and graph of the information shown on the next two pages is sorted in the same fashion. This data is "For Information Only" since Duke Energy has not performed an owner acceptance review.

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Input Parameters and Output Elevations
For Information Only

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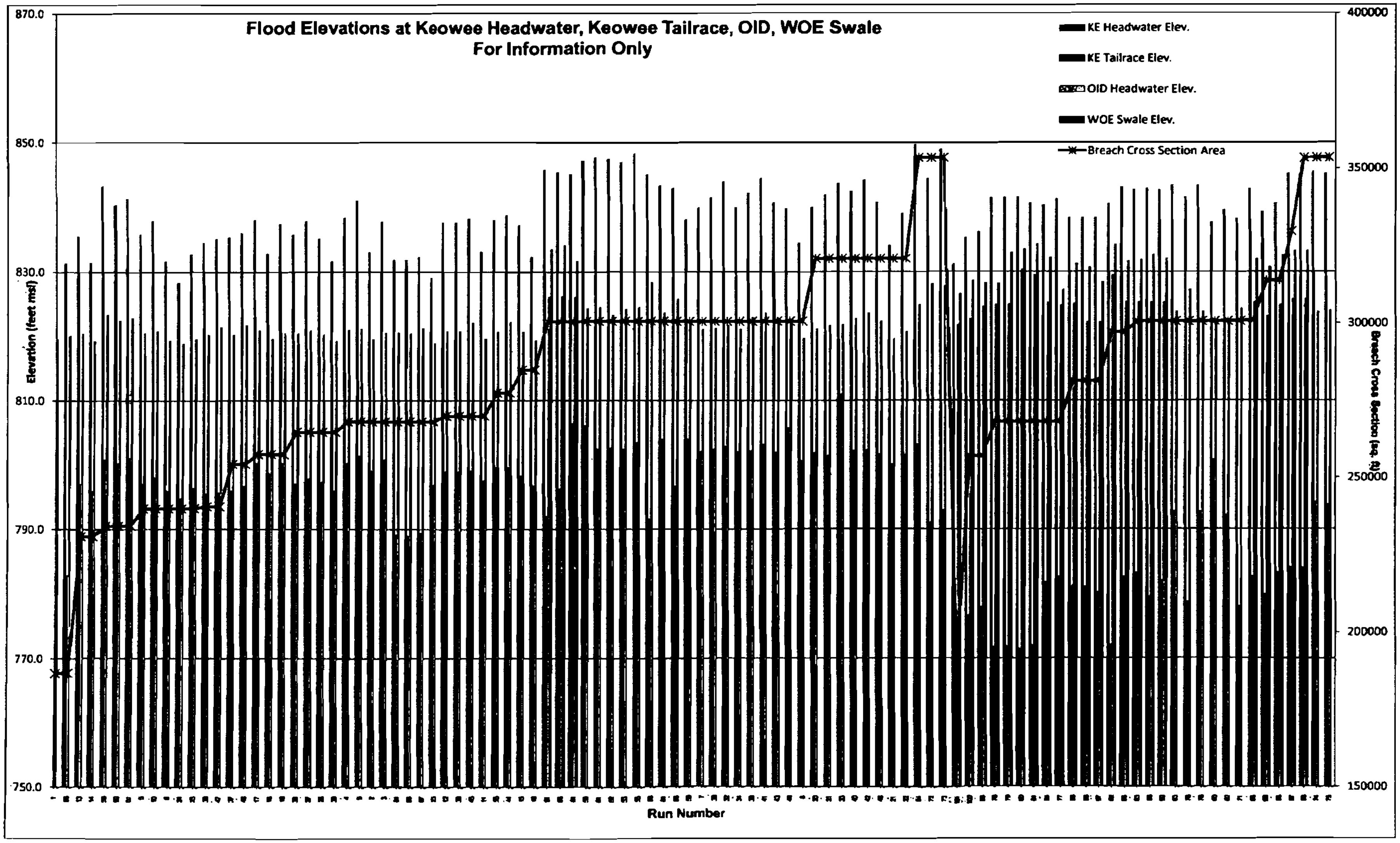

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7. **Question:**

Provide a copy of the final HEC-RAS models and 2-D models that were used for the runs to justify the proposed modifications that will be made to protect the Oconee site from external flooding.

Response: (interim)

This RAI makes reference to 'the final HEC-RAS models and 2-D models that were used for the runs to justify the proposed modifications.' Duke Energy understands this reference to mean the HEC-RAS runs and 2-D models which will be used in assessing the need for potential site modifications. More specifically, the subject models include the input files that characterize the physicality of the Keowee-Toxaway Project (Jocassee, Keowee, and the Oconee Nuclear Station) and downstream as it applies to the postulated failure of Jocassee Dam. Availability of these models is dependent upon the establishment of a common understanding of the parameters that define the case that will provide the basis for assessing the need for potential site modifications. Duke Energy intends to maintain open communications with the NRC Staff regarding the additional sensitivity cases selected and results obtained to achieve this common understanding, as such, a firm date for availability the models has not been established.

8. **Question:**

Provide a copy of the topology associated with the area below Keowee Dam and around the Oconee site yard, and switchyard.

Response:

A paper copy of the topology for the area requested is included as Attachment 2. Also, an electronic file in Adobe format has been prepared and is included in this response as Enclosure 1. The extent of the information includes the topography of the entire Oconee site, immediate earthen structures in the vicinity of the site where the parameters have been varied, and the Keowee Dam downstream tailrace area. In summary, this file contains those attributes key for the 2-D model assessments for the Oconee site and meets the scope of interest for this RAI.


ATTACHMENT 2
March 5, 2010 NRC Response
Oconee Nuclear Station
[redacted] under 10 CFR 2.390 (d)(1).
N